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FINAL REPORT
SOILS INVESTIGATION
ELLIOTT BAY INTERCEPTOR, SECTION 8
SEATTLE, WASHINGTON

INTRODUCTION

This letter and attachments summarize the final results of the soils investigation conducted along the proposed route of Section 8 of the Elliott Bay interceptor in Seattle, Washington. These results were discussed with our design engineers during the course of the investigation.

As shown on the Vicinity Map, Plate 1, Section 8 connects the Interbay pumping station on the south near Garfield Street and the existing North Trunk to the north under the Emerson Street Viaduct. It consists of approximately 5,000 feet of 96-inch diameter gravity sewer with invert varying from elevation 109* (at the north end) to elevation 111 (near station 51) and 2,900 feet of 48-inch twin force main with invert at about elevation 102. Approximately 1,700 feet of the 96-inch sewer will be constructed as tunnel. Also included in this work is an 11 x 33 foot rectangular transition structure connecting the force mains and the gravity sewer as well as a junction structure at the existing North Trunk sewer.

*Elevation 100 = Mean Sea Level

The proposed alignment is located in a filled old open-ended valley extending from Smith Cove to Salmon Bay. For about the southerly two thirds of its length, the alignment parallels the Great Northern and the Northern Pacific railway tracks. Northerly, the alignment leaves the valley flat and runs along the east side of the valley through a commercially-developed area. The existing ground surface elevation along the 96-inch gravity sewer section ranges from 40 feet above invert at Dravus Street to about one foot below the invert near Wheeler Street. Between Barrett and Wheeler Streets the existing ground surface rises sharply to the east due to garbage fill. Along the twin force main section the existing ground surface is flatter and is about 10 feet above the invert.

SUBSURFACE CONDITIONS

The subsurface conditions along the alignment were investigated by drilling 18 test borings at the locations shown on the Vicinity Map, Plate 1. A description of the field explorations and laboratory testing, together with the boring logs and test results, is contained in the Appendix.

The test borings disclosed that, in general, the proposed alignment is blanketed with a layer of surface fill ranging from 8 to about 20 feet in thickness. At the north end of the alignment, the fill contains debris and is immediately underlain by soft to moderately firm alluvial deposits which in turn are underlain at relatively shallow depths by predominately medium dense to dense sandy soils. These firmer soils were encountered at progressively higher elevations to the south (elevation 95 in B15 and elevation 112 in B4) and in the vicinity of West Bertona Street (B3) firm to very firm overconsolidated interbedded clay, silt and fine sand was encountered from the existing ground surface to the depth explored. South of West Dravus Street, surface fill was again encountered to varying depths and ranged from a relatively clean sand to a heterogeneous mixture of sand, silt, peat and debris. Near West Dravus Street, medium dense to dense sandy soils underlie the fill. Along the southerly two thirds of the alignment, the surface fill was underlain by compressible Recent marine sediments consisting of loose silty sands, soft silt and some peat with wood and shell fragments. These compressible soils extend down to elevation 108 in B16 and elevation 95 in B8 at the north end of this section and deepen southerly to about elevation 30 in B18. Medium dense to dense sands were encountered below the Recent sediments except in B12 where firm clay was observed.

As shown on the boring logs, the ground water level was observed to range from about sea level to elevation 126 along the alignment. Generally the observed ground water table was lower near the south end and ascended gradually northwards. Near the north end of the alignment a ground water level of elevation 118 was measured - approximately 4 feet higher than the controlled water level in nearby Salmon Bay.

It is believed that the subsurface conditions disclosed by the borings represent the range of soil conditions which will be encountered along the alignment. Since part of the alignment crosses an alluvium-filled valley, care should be exercised in interpolating specific soil conditions between widely spaced borings. Variations in thickness and elevation of the individual strata should be expected.

DISCUSSION AND RECOMMENDATIONS

General

The construction of Section 8 of the Elliott Bay interceptor is feasible on the basis of the information developed during this investigation. Cognizance must be taken of the ground water and soil conditions both in the design and the construction of the planned facilities. Consideration must also be given to adjacent existing construction as well as possible future development along portions of the alignment.

In the vicinity of the North Trunk connection, soft soils unsuitable for pipe support extended for some 15 feet below pipe invert. Therefore, it is recommended that piling driven into the underlying denser sandy soils be used to support the new construction from the North Trunk to about station 2+40. Near that station, satisfactory supporting soils were encountered at shallower depths. The softer soils should be excavated down to the firm soils and replaced with compacted granular backfill to provide a transition from the pile section to a soil-supported section. It is believed that the pipe may be supported on the existing natural soils at invert level southerly from about station 3+50 to the north tunnel portal at station 7+75. Between stations 7+75 and 23+25, the sewer will be constructed as tunnel to eliminate deep trenching as well as to maintain access to the adjacent industrial establishments. This section is discussed separately below.

Southerly from about station 29, soft and/or loose soils generally underlie the invert and are considered inadequate for pipe support. Further, ground settlement is still occurring north of West Wheeler Street due to the weight of the adjacent garbage fill. Along the twin force mains, it is reasonable to expect that future development which would involve dewatering or additional filling will take place during the anticipated life of the sewer. In view of the thickness and compressible characteristics of the underlying soft soils substantial settlements are also possible along that portion of the alignment. Therefore, it is recommended that the 96-inch gravity sewer, the transition structure and the twin-force mains between station 29 and Garfield Street be supported on piling driven into the deeper medium dense to dense sandy soils.

Dewatering

Consideration must be given to the proper control of ground water where excavation is required below the water table. In the pile-supported sections, south of West Barrett Street, ground water was observed near the invert level and fairly permeable sandy fill (in places some garbage and peat) was encountered during the drilling. It is believed that shallow sumps may be adequate for drawing the water table down to permit concrete pouring and backfill compaction. North of West Barrett Street, the water level was observed at or above the top of the pipe. Along this portion, shallow well points installed in a trench or within the tunnel, or deeper eductor well points installed from existing ground surface, may be necessary. In any event, any dewatering method should be designed and operated so as to prevent removal of natural soils along with the ground water. Further, dewatering should be accomplished in such a manner that settlement of adjacent property or structures does not occur.

<u>Excavation Supports</u>

Care must be taken to maintain the stability of trench excavations in view of the soil conditions encountered along much of this alignment. Adequate provisions should be included in the design of shoring or sheeting for the effect of trench-side construction operations, adjacent traffic loading and foundation pile driving. Along the trench sections the existing soils to be excavated are mostly loose fill and Recent sediments of low structural strength. Sheeting or soldier beams with lagging will be needed for the support of vertical trench excavation in those materials. Southerly, in the force main section where trench slopes are limited by a confined working area and/or adjacent buildings, trench supports may also be needed. North of station 7+75, it will be necessary to excavate within 15 feet of, and some 7 or 8 feet below, an old existing brick sewer. Care must be taken during the construction to prevent damage to this installation.

Where compacted backfill is required, any supports located below the level being compacted should be removed prior to performing compaction. Care must also be exercised during the extracting of any trench supports to prevent the sudden imposition of additional pipe loading due to the weight of backfill above the pipe.

Piling

The piles will have to support a downdrag load in addition to the structural and backfill loading. Therefore, design loads should be limited to 30 tons per pile if timber piles are used. It is recommended that the pile tips be at least 8 inches, but not more than 10 inches in diameter. Piles driven into the sand and gravel stratum to an indicated 30-ton capacity as computed from the Engineering News formula will develop adequate capacity to support the imposed loads.

On the basis of data available, it is believed that the blow counts necessary to penetrate to the sandy supporting soils will generally not exceed approximately 10 to 20 blows per foot with a Vulcan No. 1 pile hammer. However, it is possible that higher blow counts will be required in some instances. For that reason, the pile-driving must be carefully observed by a qualified inspector to avoid damage to the pile. Since it is necessary to obtain the recommended penetration in order to avoid detrimental settlement, modified methods of installing the piling such as jetting or spudding may have to be resorted to if debris in the surface fill interferes with the driving. It is believed that these modified methods, if required, would be limited in extent. In any event, the driving sequence should be such that the piles will be driven at each bent before beginning the adjacent bent.

The piling will extend above the ground water level along a short part of the pile-supported sewer. Pile cutoff will be within 3 or 4 feet of the observed ground water level along much of the remainder. However, future development of the area could result in a lowering of the general ground water table below the pile tops along the sewer. For this reason, treated timber piles or composite piles should be used throughout.

Backfill

Where subsequent backfill settlement can be tolerated or low backfill weight is desired, trench excavated material (except for debris) may be used and compaction obtained by means of jetting or comparable method. Where it is desirable to minimize subsequent settlement, it is recommended that a granular backfill material such as Type B be used and compacted by either jetting or with compacting equipment.

Within street rights-of-ways and private road areas where subsequent backfill settlement must be restricted, it is also recommended that a Type B granular material be used for backfill. The material should be placed in 8-inch thick loose lifts and compacted with mechanical equipment to 95 per cent of the ASTM D698 maximum dry density up to within 2 feet of final subgrade. To provide an adequate subgrade, the upper 2 feet should be compacted to 100 per cent dry density.

As an alternate to the latter method, Type A granular material, having zero per cent passing the #100 sieve, may be used. The material should be placed in loose lifts of 4 feet or less, then saturated or flooded and compacted by repeated injections of mass-concrete-type vibrators to 70 per cent relative density as determined by U. S. Bureau of Reclamation Designation E12B, Relative Density of Cohesionless Soils.

Between Barrett and Wheeler streets where the alignment is located at the toe of former garbage dump slope, backfill below the toe level should receive some compaction to aid future slope stability. Further, it is recommended that the disturbed slope caused by construction operations be restored at not steeper than 1-1/2 horizontal to 1 vertical. Where part of the pipe will be exposed, fill should be brought up evenly at both sides of the pipe and adequate drainage provided along each side.

Pipe Loads

The load which will be imposed on the pipe is dependent upon the subsurface conditions, height of cover above the pipe, the width of trench, the method of installation, the type of foundation support and the backfill properties. These factors were evaluated in estimating the loads the pipe would be subjected to along this alignment. For much of the alignment, the sewer pipes will be installed at shallow depths and the pipe loading will not change appreciably with varying trench conditions. It should be pointed out that the loads for vertical trench conditions would be exceeded should the trench be wider than that specified or adjacent slope failure occur. On the basis of the recommendations discussed in this report, the pipe loads due to backfill have been computed and are shown on Plate 2. Plate 3 gives the loads that must be added to the soil load where the pipe is located beneath a street (H-20 load) or railroad tracks (E-72 load).

Tunnel Section

On the basis of the borings drilled along the tunnel alignment, the heading will encounter soils which include dense fine sand, firm to very firm interbedded clay, silt and fine sand as well as man-made fill. The extent of these various soils is not known but mixed faces should be anticipated. Based on the ground water conditions observed in the borings, it is concluded that ground water control will be necessary to construct the tunnel. In view of the generally low heads observed, it is possible that well points, either installed from the surface or within the tunnel, may suffice. Among other possible methods is soil solidification in advance of the face to avoid dewatering.

In view of the soil and ground water conditions anticipated at the tunnel level, continuous support of the ground will be necessary until the permanent concrete lining is placed. While the firm silts and clays, except if sand layers or lenses are encountered, will stand for a sufficient period of time to permit installation of tunnel supports behind the face, it is believed that the water-bearing sands and man-made fill will require immediate support to prevent loss of ground. Therefore, the use of a shield to advance the tunnel and provide ground support during tunnel support erection is recommended. In the firm clays, it may be necessary to mine at or ahead of the cutting edge since the shield probably could not be forced into the material for the full shove. A full circular section heading is recommended.

Care must also be exercised during mining and dewatering, if used, to prevent damage to existing adjacent structures. Several of the buildings along the west side of the tunnel alignment are supported on foundations above the tunnel level. These include the structures at 3209, 3257 and 3443 Thorndyke Avenue West. Examination of plans on file at the Seattle Building Department indicate that these structures have footings founded as high as elevation 130±. The remaining structures are either supported below tunnel level or plans were not available.

The external pressures which are imposed on both the permanent and temporary (construction) tunnel lining are a function of tunnel-heading dimensions, soil and ground water conditions, and construction methods. Exact analysis of these pressures is neither possible nor warranted since variable conditions and practices during construction can nullify the assumptions on which the pressure calculations are based. Therefore, empirical methods and formulas, weighted by rational analyses, are usually employed. It has been established that the lining pressure is generally substantially less than the weight of overburden due to development of arching action in the immediate overlying soils. A certain amount of strain or downward movement of the soil at the tunnel roof is required to develop the necessary shear strength, and thus allow the formation of the ground-arch. This will vary with the type and density of the soil. Little definitive data is available as to the required magnitude of this movement since part of it occurs during the mining of the heading. This, plus the initial deflection of the liner crown as the load comes on, has usually been found sufficient to allow the load reduction to take place. If the required amount of movement is exceeded, an increase in tunnel pressure will occur. Thus, loss of ground and/or unnecessary overmining must be prevented or immediate filling of the voids by backpacking and/or grouting is necessary to minimize the effect.

Analyses have been made of the external loadings which will be imposed on the permanent concrete lining for both steel rib and wood lagging, and liner plate circular construction supports. A 12-foot diameter heading was assumed for the rib and lagging section and an 11-foot diameter heading for the liner plate section. When a circular liner plate section is used, the liner plates function as a compression ring since they are incapable of resisting moment. Thus, while the initial loading would be similar in magnitude to the rib and lagging section, the unbalanced loading will deflect the liner plate crown and cause a shortening of the vertical diameter with a resulting increase in horizontal diameter. This will create a condition of passive pressure at the sides and cause a higher horizontal loading which will tend to equalize the peripheral loading. This will occur prior to installing the permanent lining and the permanent circular lining can be designed for equal all-around pressures.

The following permanent lining pressures are estimated along the tunnel alignment:

Method	Dewatered Vertical PSF	Dewatered Horizontal PSF	Maximum Ground Water* Vertical PSF	Maximum Ground Water* Horizontal PSF
			W.L. = El. 126	
Liner plate	2850	2850	2900	2900
Lagging	3100	1150	3150	1750

Note - Use liner plate values only if properly backpacked.

*Includes hydrostatic pressure vertical = above crown
horizontal = above spring line

If a larger heading is driven than that assumed, or loss of ground occurs, a higher lining pressure will result. In either event, the recommended loading should be revised.

The lowest observed ground water level along the tunnel was at about elevation 116 (B4); thus it appears that part of the tunnel may not be below the permanent water table. Should wood lagging be used for the temporary lining support, it is recommended that all the voids around the permanent lining be thoroughly grouted so that possibility of inducing unbalanced lining pressure due to lagging decay will be minimized.

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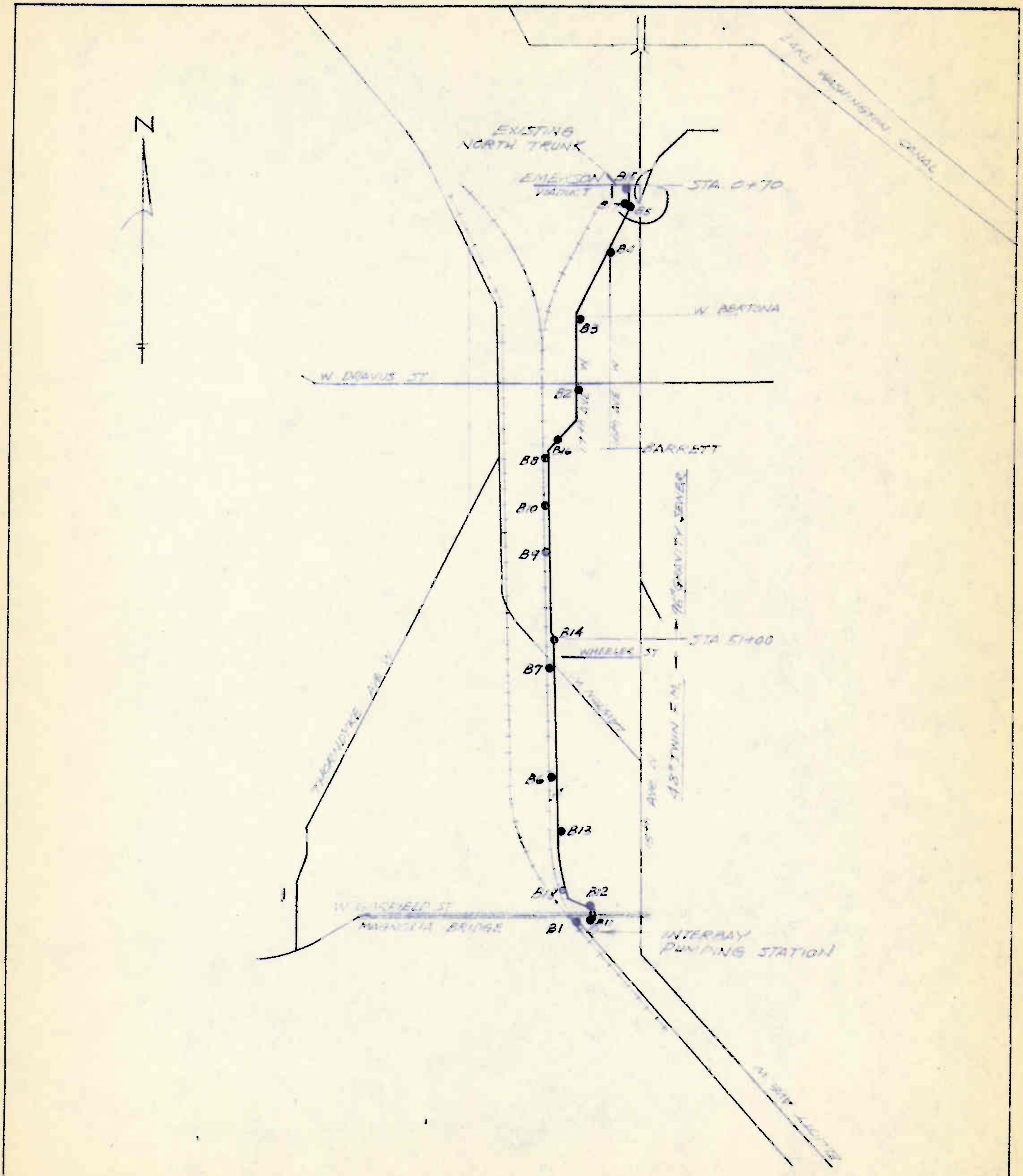
Attachments:
Plate 1 - Vicinity Map
2 - Pipe Loads
3 - H-20 and E-72 Loading
Appendix

CALCULATION SHEET

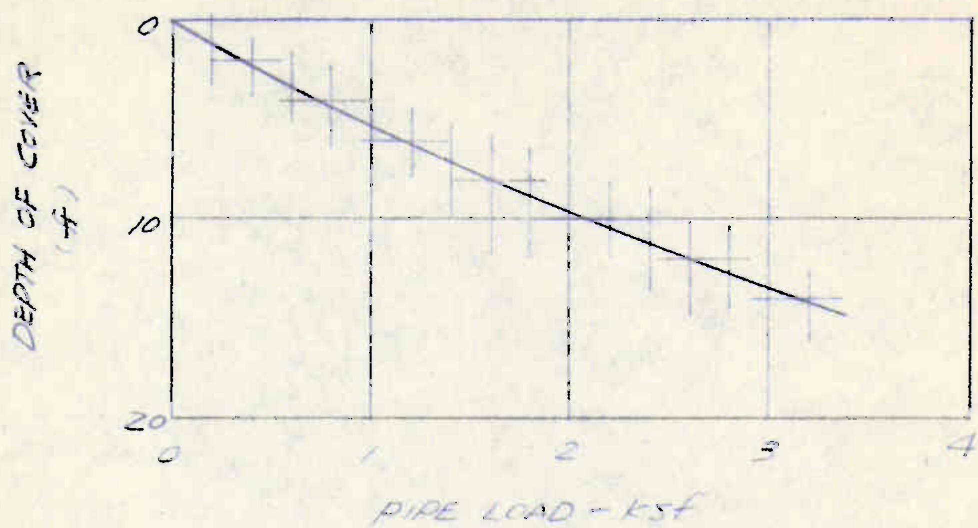
METROPOLITAN ENGINEERS
SEATTLE, WASHINGTON



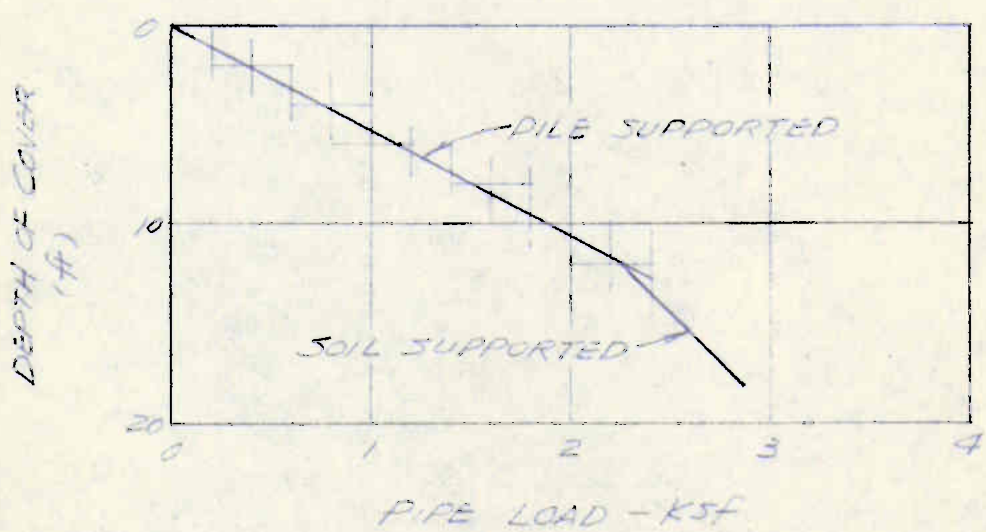
DATE	BY	JOB NO	TITLE	PLATE
		W210D	VICINITY MAP	1

CALCULATION SHEET

METROPOLITAN ENGINEERS
SEATTLE, WASHINGTON



TWIN 48"ϕ FORCE MAINS PIPE LOADS



96"ϕ PIPE LOAD

DATE	BY	JOB NO.	TITLE	PLATE
	CHW	W210D	PIPE LOADS	2

CALCULATION SHEET

METROPOLITAN ENGINEERS
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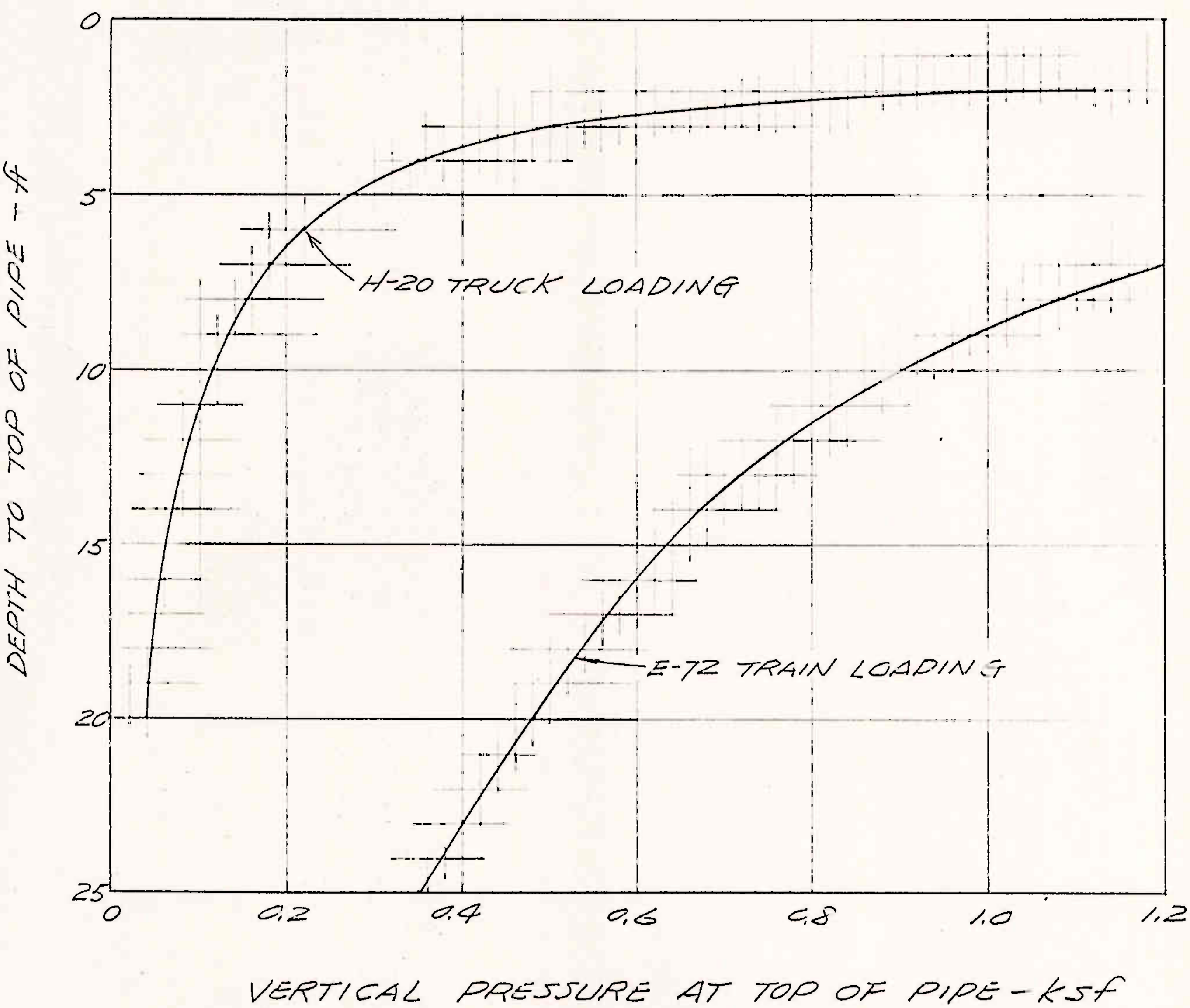

DATE	BY	JOB NO.	TITLE	PLATE
	CHW	W210D	H-20 & E-72 LOADING	3

Appendix

FIELD EXPLORATIONS AND LABORATORY TESTING

Field Explorations

The subsurface conditions along the alignment were explored by the drilling of 18 test borings at the locations shown on the Vicinity Map. The locations, with reference to stationing and offset from the sewer center line, are also noted at the top of each boring log.

Borings B1 through B8 were drilled with truck-mounted cable-tool drilling equipment. Representative undisturbed samples of the soils encountered were secured for visual examination and laboratory testing. These samples were taken using a 3.25-inch O.D. split-barrel sampler driven by means of a 500-pound drop weight falling approximately 20 inches. Borings B9 through B18 were drilled with 7-inch O.D. hollow-core continuous flight auger drilling equipment and only disturbed samples were taken using the standard penetration test method. This method involves driving a 2-inch O.D. split-spoon sampler by means of a 140-pound drop weight falling 30 inches.

All the borings were logged in the field by our geologist. The soils encountered in the exploratory borings, together with the highest observed water levels, are shown graphically on the boring logs, Plates A-1 through A-16. The blow counts required to drive the samplers one foot are shown at the respective sample elevations.

Laboratory Testing

Laboratory testing was limited primarily to determining the natural in-place densities and moisture contents of the representative samples. The results of these tests are shown on the respective boring logs. Sieve analyses were also run on certain samples as an aid in evaluating dewatering characteristics; the resulting grain-size curves are presented on Plates A-17 through A-21. It should be pointed out that the sieve tests were performed on samples taken with 2-1/2 inch and 1-3/8 inch I.D. samplers. Thus material larger than these dimensions may be present in the soils although it is not indicated on the gradation curve.